Concrete bridge deck slabs strengthened with UHPFRC

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Summary

An original concept is presented for the durable rehabilitation of concrete bridge deck slabs. The main idea is to add a layer of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) with steel reinforcing bars over the concrete slab to create a composite section. The layer of UHPFRC is waterproof and protects the reinforced concrete against severe environmental influences. It also strengthens the structural element for high traffic loads. Experimental studies on composite beams in a cantilever setup were carried out to identify the different failure modes and the contribution of the UHPFRC layer to the resistance. Analytical models were then developed to calculate the resistance of composite beams. The concept has been validated by field applications demonstrating that the technology of UHPFRC is mature for cast in-situ.

Keywords: UHPFRC, composite section, reinforced concrete, resistance, failure mode, analytical model, rehabilitation, strengthening.

1. Introduction

Rehabilitation of deteriorated concrete structures is a heavy burden from the socio-economic viewpoint since it leads to significant user costs. As a consequence, novel concepts for the rehabilitation of those structures must be developed. To increase the life span of a reinforced concrete bridge deck slab, it is possible to add a layer of 30 to 60 mm of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) with or without small diameter steel reinforcement bars, thus creating a composite section (Fig. 1).

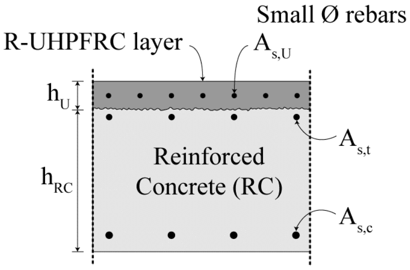

Fig. 1: Typical composite section [3]

UHPFRC is a cementitious composite material known for its excellent properties: extremely low permeability which prevents the ingress of detrimental substances such as water and chlorides [1] and very high strength, i.e., compressive strength higher than 150 MPa, tensile strength higher than 10 MPa and with considerable tensile strain hardening and softening behaviour [2]. A UHPFRC layer can thus be used to protect the concrete slab against severe environmental influences and to strengthen it for high traffic loads as proposed in [3]. In this paper the layer of UHPFRC is considered as a tensile reinforcement for the reinforced concrete (RC) section

The original conceptual idea (developed in 1999) has been investigated by means of extensive research aimed at characterizing UHPFRC as well as the structural behaviour of R-UHPFRC – RC composite structural members, combining material and structural engineering sciences. This paper presents the tensile behaviour of UHPFRC followed by the results of a large experimental study on a series of composite beams tested in a cantilever test setup. Using the test results, analytical models to predict the failure mode and resistance of the composite beams are developed. To conclude a field application on a bridge in Switzerland is described.

2. UHPFRC in tension

As illustrated in Fig. 2, the uniaxial tensile behaviour of UHPFRC is divided into three phases. First, the material is elastic up to initiation of microcracking of the matrix. The elastic limit strength $f_{Ut,el}$ has typical values of 7 to 11 MPa for currently used UHPFRCs.

Second, it goes into a phase of strain hardening with multiple (non-visible) microcracking of the matrix and fiber activation. The material still behaves like a continuum. Strain hardening domain may reach strains of 2 to 5‰ where the tensile strength $f_{Ut,u}$ is reached with typical values ranging from 9 to 15 MPa.

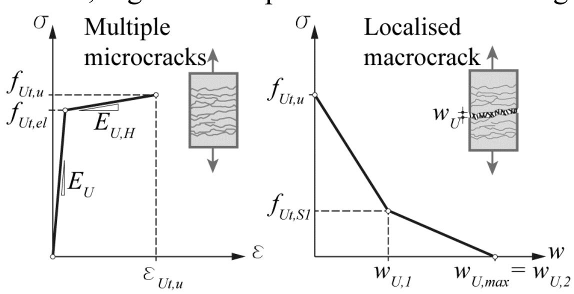

Fig. 2: Tensile behaviour of plain UHPFRC [3]

Third, upon the formation of a discrete macrocrack at ultimate resistance, the phase of strain softening begins and the behaviour of the material is described by a stress-crack opening law with maximum crack openings reaching about half of the fiber length, i.e., 5 to 8 mm.

3. Experimental investigation on composite beams

Flexural tests on composite beams showed that the layer of R-UHPFRC, when used as a tensile reinforcement, increases the resistance up to 165% compared to a reference RC beam [4]. A flexural failure happens along a vertical crack where the highest moment is applied. A macrocrack appears in the RC section while distributed microcracks develop in the UHPFRC layer. In the post-peak domain, fracture of R-UHPFRC occurs in the section with the localization of a macrocrack. No debonding is observed at the interface between the UHPFRC layer and the concrete prior to failure [3]. It is thus supposed that the behavior of composite beams is monolithic when submitted to pure flexural moments.

To study the behavior of composite beams submitted to combined bending and shear, an extensive experimental campaign was carried out. Composite beams were tested in a cantilever test setup. The main goal of these tests is to study the different failure modes and mechanisms of composite beams and the contribution of the UHPFRC layer to shear resistance. Important results of this campaign are presented hereafter.

3.1 Specimens

Two types of specimens were tested. The beams of the B series were tested by Noshiravani [6]. As shown in Fig. 3. (a), they have a width of 150 mm and a total height of 300 mm. They are similar to slab ribs with stirrups spaced at 400 mm. The beams of the S series (Fig. 3 (b)) are wider with a width of 400 mm and a total height of 220 mm. These specimens are closer to slab strips because they have no shear reinforcement. For both cases, the layer of UHPFRC has a height of 50 mm.

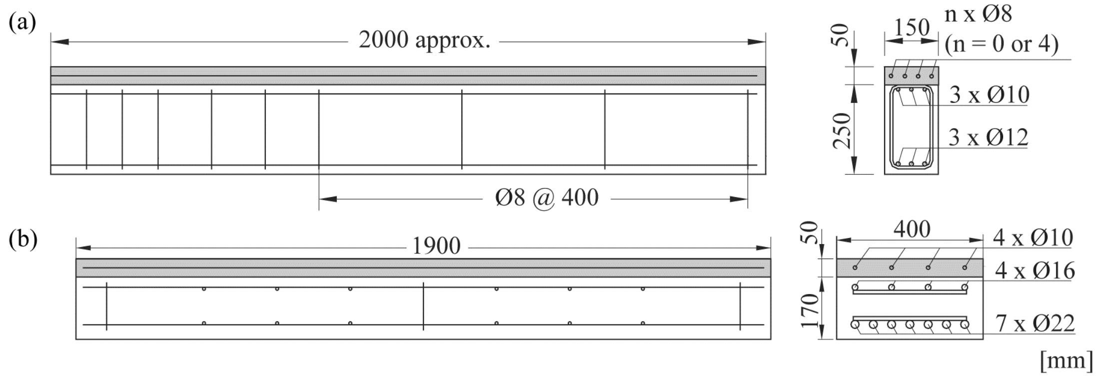

Fig. 3: Geometry of specimens (a) B series [6]; (b) S series

3.2 Material properties

Table 1: Concrete and UHPFRC properties

	Concrete		HIFCOM 13				
Series	E_c [GPa]	f_c [MPa]	E_c [GPa]	f_c [MPa]	$f_{ut,el}$ [MPa]	$\varepsilon_{ut,u}$ [‰]	$f_{ut,u}$ [MPa]
B [4]	29,9	41,6	48,8	160	10	3,0	12
S	31,4	56,8	43,8	227	10	3,0	12

All RC substrates were fabricated with a normal strength concrete and a maximum aggregate size of 16 mm. The layer of UHPFRC is cast with an in-house mix called HIFCOM 13 [4]. Table 1 gives the average values of the concrete and UHPFRC properties at the moment of testing.

Table 2: Steel properties

Ø [mm]	f_{sy} [MPa]	ε_{su} [‰]	f_{su} [MPa]
8	516	4,9	589
10	594	4,3	653
12	571	5,0	640
14 and more	565	9,8	663

The steel used in the RC section and in the R-UHPFRC layer has nominal yield strength of 500 MPa. Table 2 gives the average values obtained from standardized tensile tests.

3.3 Test setup and parameters

All specimens were tested in a cantilever test setup, as shown in Fig. 4. The shear length, *a*, varies between 450 mm and 1000 mm. The length *l* between the load P and the pin support is 1600 mm for the B series and 1500 mm for the S series. The external vertical prestressing between the roller and the pin prevents a shear failure outside the cantilever span.

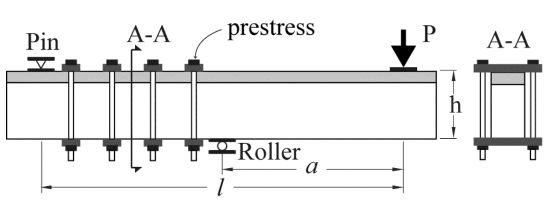

Fig. 4: Cantilever test setup

Table 3 gives the main test parameters for every specimen presented in this paper: the shear span *a*, the ratio between the span and the equivalent static height *a/d*, the amount of shear reinforcement ρ_{sv} and the mechanical reinforcement ratio ω_i. In this case, the subscript *i* stands for the tensile longitudinal reinforcement which are UHPFRC (*U*) or tensile rebars in R-UHPFRC (*s,U*) or RC section (*s,t*). Equations 1 and 2 calculate *d* and ω_i where d_i is the static height of the considered reinforcement, A_i is the area and f_i is the tensile strength.

$$d = \frac{\sum d_i A_i f_i}{\sum A_i f_i} \tag{1}$$

$$\omega_i = \frac{A_i f_i}{A_c f_c} \tag{2}$$

Table 3: Test parameters

Series	b [mm]	h [mm]	Beam	a [mm]	l [mm]	a/d	ρ_{sv} [%]	$\omega_{s,t}$ [%]	ω_U [%]	$\omega_{s,U}$ [%]
B [6]	150	300	B_MW0	800	1600	3,4	0,17	8,1	0	0
			B_MW1	800	1600	3,2	0,17	8,1	3,9	0
			B_MW4	800	1600	3,1	0,17	8,1	3,9	7,3
S	400	220	S_L1	1000	1500	6,0	0	12,3	6,2	4,8
			S_M1	700	1500	4,2	0	12,3	6,2	4,8
			S_S1	450	1500	2,7	0	12,3	6,2	4,8

The tests are displacement controlled. The displacement is applied with a hydraulic jack and a load cell measures the force, while a linear vertical displacement transducer measures the displacement.

3.4 Test results

3.4.1 B series [6]

To show the contribution of the layer of UHPFRC to the failure mechanisms and resistance of a composite beam, the results of tests conducted on three specimens of the B series are presented hereafter. Each specimen has a different amount of tensile reinforcement. Beam MW0 is a reinforced concrete beam with no UHPFRC layer. Beam MW1 has a plain layer of UHPFRC and beam MW4 has a layer of R-UHPFRC. All the beams have a shear span of 800 mm.

Fig. 5 presents the test results for those three specimens. The RC beam, MW0, failed in flexure-shear. A flexure-shear failure occurs when a flexural vertical crack rotates towards the roller support and develops diagonally. At maximum force, sudden decrease of resistance was observed followed by some deformation capacity on a rather constant force level. Fracture finally was due to the crushing of the concrete near the support at the crack tip.

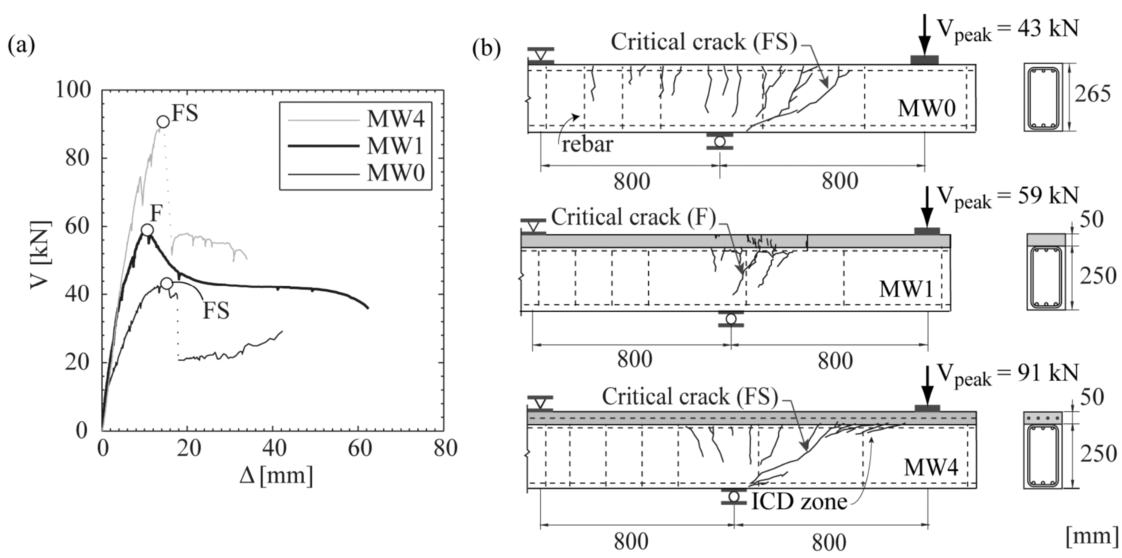

Fig. 5: Tests results for B series [6]: (a) Force-deflection responses; (b) crack patterns at peak load

By adding a layer of plain UHPFRC over the RC section (beam MW1), no sudden decrease of resistance after peak occurred and the resistance is increased. Beam MW1 showed a flexural failure along a vertical crack over the support. After a ductile flexural failure, the element retains a significant proportion of its resistance and rotation capacity. In the case of beam MW4, reinforcement was added in the UHPFRC layer. The resistance of the element was once more significantly increased. The beam failed in flexure-shear, showing relatively small deformations. This demonstrates that it is possible to over-strengthen a beam in bending thus leading to a flexure-shear failure. The cracking pattern of beam MW4 gives interesting information on the shear resisting mechanisms of a composite beam. During the test the widening of the inclined crack caused a softening of the concrete below the UHPFRC layer. This debonding occurs between the mouth of the inclined crack and the load point. It is due to the opening of a diagonal crack and is known as Intermediate-Crack induced Debonding (ICD) [6].

3.4.2 S series

The results of the tests carried out on the beams of the S series are presented in Fig. 6. The three specimens were identical with varying shear span: 1000, 700 and 450 mm. In all cases final failure was flexural along a vertical collapse crack over the roller support. The sudden decrease in resistance visible in the force-deflection diagrams for every beam is due to the rupture of the reinforcing bars in the UHPFRC layer. This always happens in the post-peak domain.

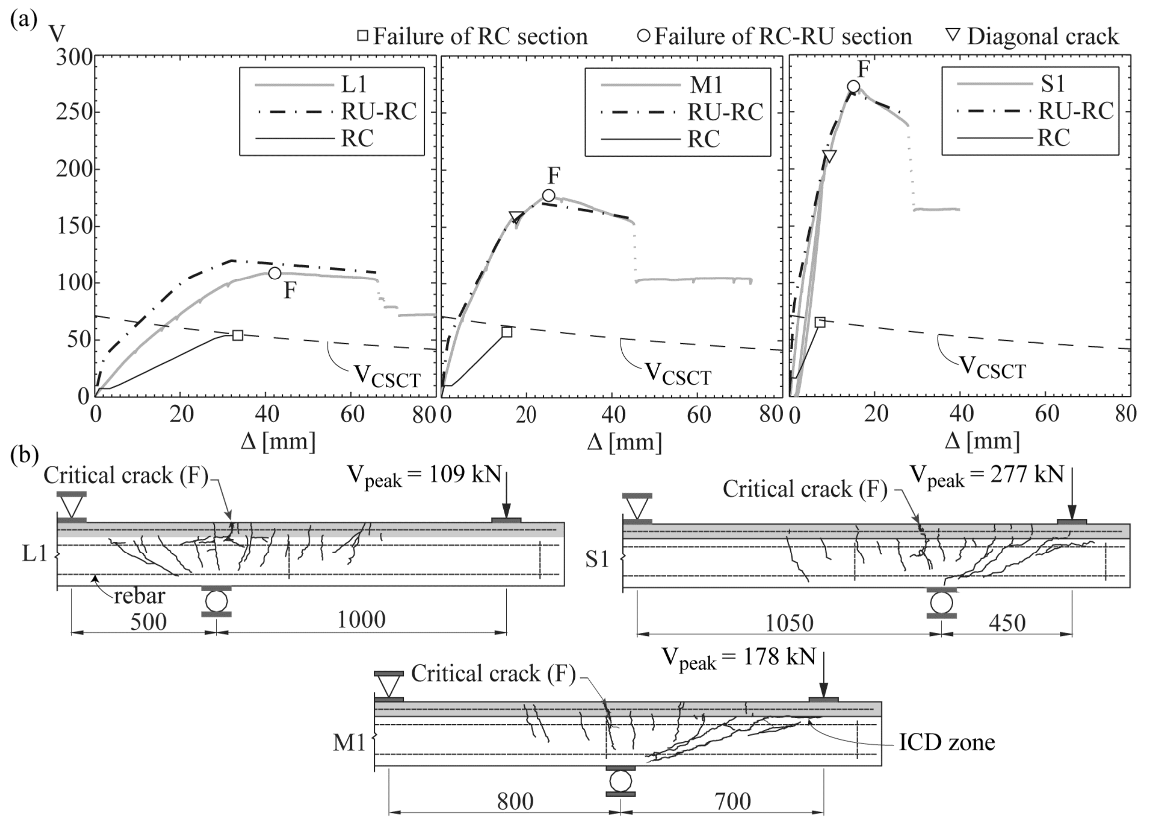

Fig. 6: Tests results for S series: (a) Force-deflection responses; (b) crack patterns at peak load

The graphs in Fig. 6 also give the force-displacement response of the RC section alone. This was calculated with a plain sectional analysis combined with a shear failure criterion. This criterion is based on the critical shear crack theory (CSCT) [7] and is calculated with equation 3. The CSCT calculates shear resistance of concrete as a function of the opening of a diagonal crack and takes account for aggregate interlock.

$$\frac{V_{CSCT}}{bd\sqrt{f_c}} = \frac{1}{6}\,\frac{1}{1 + 2\,\dfrac{\psi d}{16 + d_g}} \tag{3}$$

$$\psi = \frac{\Delta}{a} \tag{4}$$

In all cases, the RC section would have failed in shear and the layer of R-UHPFRC contributed to prevent shear failure and increase the rotation capacity. In the case of beam L1, no diagonal cracks were observed during the test. A triangle in the force-deflection graphs shows the instant when the diagonal crack appeared for beams M1 and S1. The layer of UHPFRC resisted to the opening of this diagonal crack, forcing the final failure to be flexural.

4. Analytical model

4.1 Behaviour in bending

As described previously, when a beam fails in bending, no debonding between UHPFRC and concrete occurred. The composite beams thus behaved monolithically and the hypothesis of Bernoulli - plane sections remain plane- is valid. To calculate the moment-curvature response and the ultimate resisting moment in bending for a composite beam, it was thus proposed to use a plane section analysis as given in Fig. 7 [3].

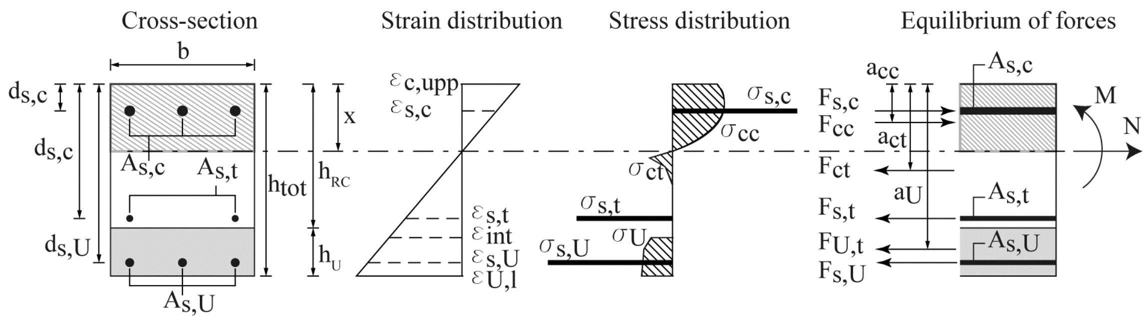

Fig. 7: Definition of the analytical model for bending [3]

For the three beams of the S series, the force-deflection response was calculated according to this model. The results of these calculations are presented on the graphs of Fig. 6 with the dash-dot line. The model is in good agreement with the experimental results.

4.2 Behaviour in combined bending and shear

As illustrated by beam B_MW4, the ICD caused the failure of a composite beam in flexure-shear. Fig. 8 illustrates the mechanism as observed before the collapse. When the diagonal crack widens, it creates a prying action on the UHPFRC layer thus creating the softening of the ICD zone and submitting the R-UHPFRC layer to a double curvature [6]. With this debonding, the section can no longer be considered as monolithic and the member stiffness decreases.

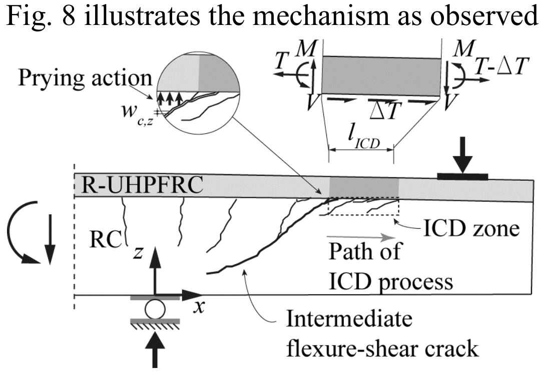

Fig. 8: Intermediate-Crack induced debonding (ICD) [6]

This mechanism is used in [6] to propose a simplified formulation to evaluate the flexure-shear strength of a composite beam, $V_{RU\text{-}RC}$. Equation 5 calculates it as the sum of the contribution of concrete (V_C), stirrups in steel (V_S) and UHPFRC layer (V_{RU}).

$$V_{RU-RC} = V_c + V_s + V_{RU} \tag{5}$$

The shear resistance of concrete V_C along the diagonal crack is obtained with Equation 6 proposed in [8]. In this equation, f_{ce} is the effective strength of concrete and can be taken as $0{,}8f_c$, c_{FS} is the height of the neutral axis in the RC section, θ_c is the angle of the diagonal crack in respect to the longitudinal axis while α is the angle of this crack with the vertical axis.

$$V_c = \frac{f_{ce}\,b}{2}\left[\frac{C_{FS}}{\sin\theta_c}(1-\sin\alpha)\right] \quad (6)$$

$$V_s = A_v f_{sy} \quad (7)$$

The shear resistance of the UHPFRC layer submitted to bending in double curvature is a function of its flexural resistance $M_{RU,max}$ and the length of the ICD zone $l_{RU,dl,max}$, as stated by equation 8. The bending resistance can be calculated with a plane section analysis while the ICD length is obtained geometrically using the angle of the diagonal crack θ_c and the span length a.

$$V_{RU} = \frac{2M_{RU,max}}{l_{RU,dl,max}} \quad (8)$$

4.3 Model validation

Table 4: Model validation

	S_M1	B_MW4
Parameters		
C_{FS} [mm]	55	90
θ_c	25	30
α	65	60
$M_{RU,max}$ [kN.m]	8	4
$l_{RU,dl,max}$ [mm]	235	280
Strenght		
$V_{R,f}$ [kN]	**171**	95
V_c [kN]	111	60
V_s [kN]	0	0
V_{RU} [kN]	67	25
$V_{R,shear}$ [kN]	177	**86**
V_{peak} [kN]	178	91

To validate the two models presented in the previous sections, the results of the calculations for two tested beams were compared to the experimental results (V_{peak} in Table 4). The two chosen beams are B_MW4 and S_M1. In both cases, an ICD zone was observed during the test (Fig. 6 and 7). For S_M1, the failure was in bending and for B_MW4, it was in flexure-shear. The resisting moment in bending was converted into a force ($V_{R,f}$) by dividing it by the shear span a.

In Table 4, the value in bold is the minimum between the shear and the flexural strength. It determines the failure mode. The analytical models were able to predict with a good precision the failure mode and the strength of the considered beam (measured experimentally). It is also interesting to notice the little difference between the shear and the bending strength for both cases illustrated here. This shows the importance of a good design for the UHPFRC layer to guarantee ductile behavior of the member.

5. Application

A RC massive slab bridge supported by 6 columns, built in 1963 and located near Lausanne, in Switzerland, was reinforced with a layer of UHPFRC (Fig. 9) in autumn 2011 [9]. Before the intervention, the slab parts over the column supports did not meet the requirements for structural safety in bending and shear of the Swiss standards for existing structures [10]. The top layer of approximately 20 to 40mm of the RC slab was first removed with high pressure water jet. Then, a layer of 25 mm of UHPFRC was cast over the slab for strengthening and waterproofing the slab. Over the column supports, a thicker layer of 65 mm was placed with 18 mm diameter steel rebars. This intervention significantly improved the load bearing capacity and durability of the bridge. It also demonstrated the simplicity in applying this technology.

Fig. 9: Application case

6. Conclusion

An experimental campaign on composite elements has demonstrated that a layer of R-UHPFRC over a RC section significantly increases the load bearing capacity and prevents shear failure if designed correctly using the proposed analytical models. A site application showed that this concept is also simple to apply to full size structures.

This conceptual idea combines efficiently the protection and resistance properties of UHPFRC with conventional structural concrete. The rehabilitated structures have significantly improved structural resistance and durability. This original concept should also be applied for the construction of durable new reinforced concrete structures [11].

7. References

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